Wedge analysis for adverse geological reaches of pump house heading portion –a case study

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Abstract

A 51 m deep underground pump house cavern has been excavated for lift irrigation purpose in Telangana State, India. Excavation of pump house cavern was done with pilot tunnel 8 m wide and 9 m high. Pilot tunnel excavation methodology is technique helpful for assessment of rock mass quality, excavation time, planning for rock support and treatment of geological features if any during the side slashing and bench down time. A sub horizontal joint and a sub vertical joint with persistence more than 50 m were encountered at the crown level during the pilot tunnel excavation. Apart from design rock support installation, Unwedge analysis was done in these reaches for adverse joints with combination of other surrounding joints. Potential wedges were predicted and requirement of additional rock support was followed as per Unwedge software. In this paper wedge analysis for adverse geological joints of pump house heading portion is discussed.

1. Introduction:

Kaleshwaram project, has been conceived from Dr. B.R. Ambedkar Pranahita-Chevella-Sujala-Sravanthi project. Originally, Dr. B.R. Ambedkar Pranahita-Chevella-Sujala-Sravanthi project was proposed to utilize 160 TMC of allocated water of Godavari basin as per GWDT award. A barrage was proposed at Tummidihetti (V) to divert 160 TMC of water to irrigate 16.40 lakh Acre lands in 7 districts of Telangana State, India viz., Adilabad, Nizamabad, Karimnagar, Medak, Warangal, Nalgonda & Rangareddy, besides drinking water & industrial water.

The project contemplates to provide Irrigation facilities for an ayacut of 16,40,000 Acres in drought prone areas in 7 Districts in Telangana State. Further, it also provides 10 TMC of Drinking Water to the villages enroute, 30 TMC of drinking water to twin Cities of Hyderabad, Secunderabad and 16 TMC of water for Industrial use.

Package-6 of this scheme is being constructed to lift 146.24 TMC water from SripadaYellampalli reservoir to Medaram tank situated near Dharmaram Mandal in Karimnagar District. The water need to be lifted from the tunnel bed level at RL+ 109 m to Medaram tank at its FRL +230 m. Twin tunnels 10 m finished dia and 9.55 km long have been constructed to supply the water with the design discharge of 535 cumecs. Twin tunnels joined the surge pool cavern, from where draft tube tunnels leading to the pump house cavern have been constructed.

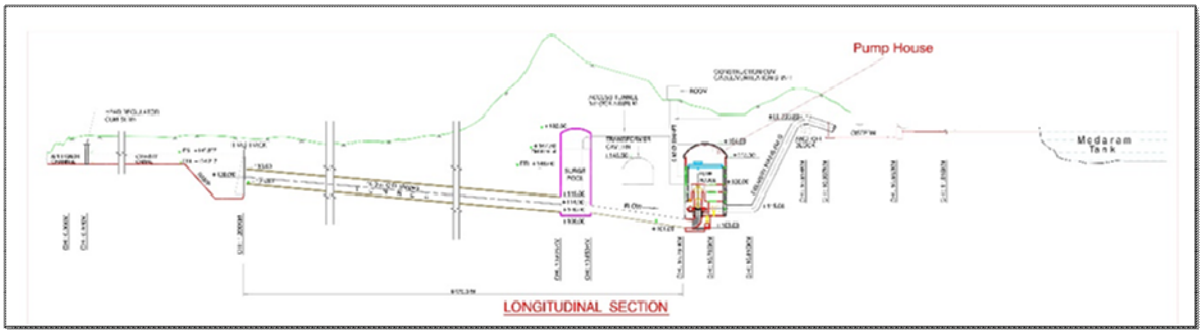

Figure 1 Longitudinal section of pump house and other components of package-6, of Kaleshwaram Lift Irrigation project

Water will be lifted from RL+ 109 m to the cistern located at ground level RL +241 m with the help of pumps and through pressure/delivery mains. Water from cistern (+241 m) to the Medaram tank (FRL +230 m) will move through gravity. To lift the huge amount of water during the monsoon high capacity of pumps (6 x 124.4 MW) are being installed in the pump house cavern. The pump house cavern is 51 m high, from EL 154.5 m to 103 m and 25 m wide (Fig 2.). The longitudinal direction of the pump house cavern is N320°.

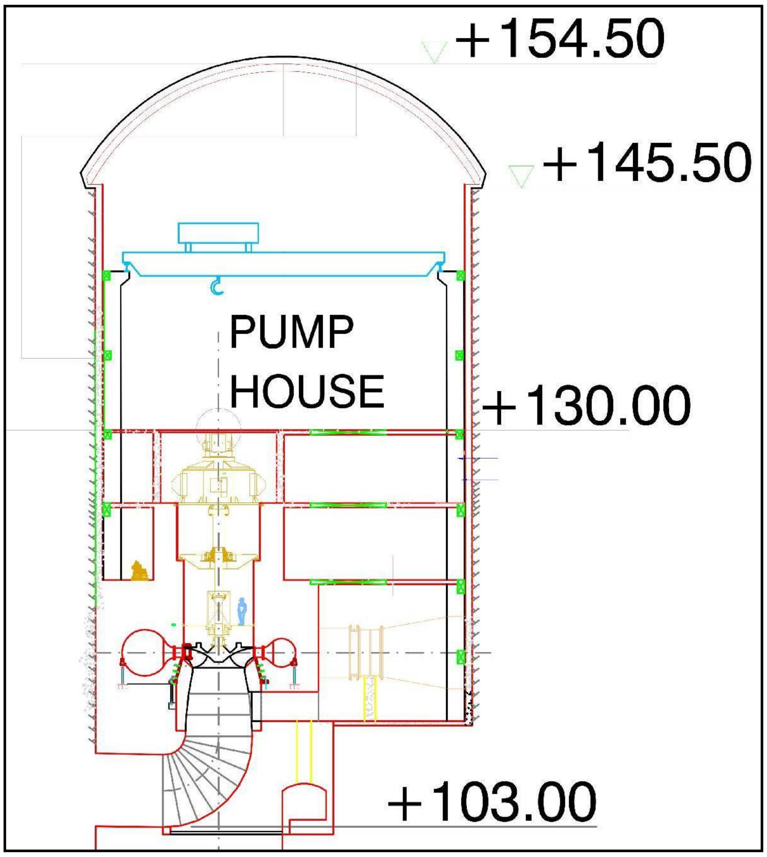

Figure 2 Longitudinal cross section of Pump house of package

2. Geology of the Area:

The regional lithological units belong to NW-SE trending Karimnagar Granulite Terrain (KGT), supracrustal rocks of Peninsular Gneissic Complex (PGC) of Archean age. Basics enclaves of older metamorphics occur within PGC granites at places which are mainly represented by pyroxinite/amphibolite. Dolerite dykes are also emplaced into the main litho units at places. Medium to coarse grained, grey to pinkish grey granites, migmatite, gneiss and charnokites intruded by dolerite dyke and basic enclaves of Karimnagar Granulite Belt are exposed in the area. The general stratigraphic sequence of the study area is given in (Table 1). The area is predominantly pediplain terrain dotted with inselberg, denudational and residual hillock developed over resistant granite rocks and small linear ridges of dolerite dykes.

Table1 Generalized Stratigraphic Sequence

Quartz/ pegmatitic veins (acid intrusive)		Peninsular Gneissic Complex
Granodiorite		
Biotite granite gneiss/ tonalite		
Amphibolites/ pyroxene granulite		High Grade Supracrustals
Banded magnetite quartzite/ quartzite /schist		

3. Excavation of pilot tunnel and adverse geological reaches:

Pilot tunnel was excavated in good to very good granite rock tunnelling media upto 95 m chainage. At chainage 95.2 m tunnel reach a sub horizontal joint N40°-80°W/5-30° was encountered at crown level upto 117 m chainage at right crown level and intersected inside the left wall at 129m. Joint S40-80°W/45-70° was mapped at 113 m and extended upto159 m at the right wall of pump house. Both the joints were encountered with filling of 1-3 mm of non softening material.

After the excavation of pilot tunnel, problems were anticipated with these joints during side slashing and bench down time. Potential wedge formation in combination with the other joints were analysed under Unwedge software.

4. Engineering geological mapping:

Geological face mapping was carried out on 1:100 scale and 3D geological logging on 1:200 scale. Pilot tunnel was excavated in medium to coarse grained granite rock. Joints are rough irregular, planar to smooth undulating in nature. Joint surfaces are stained to slightly altered in nature. Excavation was done in dry tunnelling media except at chainage 125 m where at the spring level dripping of water was observed from the existing bore hole. This existing bore hole was suggested to back fill with the grouting. Following joint sets encountered during the excavation of pilot tunnel and side slashing.

Table 2 Joint sets encountered during pilot tunnel and side slashing

Joint Set	Dip direction / Dip Amount	Spacing (cm)	Persistence (m)	Roughness*	Aperture (mm)	Infilling	Ground Water	Remark
J1	S40-80°W/45-70°	50-100	>50	R/I/P, S/U	Tight-3	Non softening	Dry	Critical joint
J2	S80°W to N40°/75-90°	20-100	>15	R/I/P, S/U	Tight-3	Non softening	Dry	Prominent joint
J3	N80°W and N40°W/5-30°)	50-300	>55	R/I/P, S/U	1-3	Non softening	Dry	Prominent joint
JR1	S40°W and S80°W/70-85°	-	>10	R/I/P, S/U	1-3	Non softening	Dry	Random joint
JR2	N70°E and N90°E/70-75°	-	>20	R/I/P, S/U	1-3	Non softening	Dry	Random joint
JR3	N40°E -50°E/5-20°	-	>10	R/I/P, S/U	1-3	Non softening	Dry	Random joint
JR4	N60°E-S80°E/75-80°	-	>10	R/I/P, S/U	1-3	Non softening	Dry	Random joint
R/I/P, S/U* Rough irregular planar, Smooth undulating								

RQD ranges between 85 to 95. Rock mass quality index Q ranges between11.07 to 49.24 which is characterized into good to very good rock type. Temporary rock bolts were installed as per site geology after the every blast.

5. Rock support of pilot tunnel:

After the excavation of pilot tunnel upto 210.6 m length one row of design rock bolts were installed at the centre of crown. Design rock bolts of 7 m length at the 2m x 2m were helpful for support the excavated rock mass.

6. Side slashing of pump house:

Excavation of side was started from chainage 85 m at left side in view of prominent joints N40°-80°W/5-30° with filling at these chainages and timely rock support was done in view of sub horizontal persistence joint. After the 25-30 m excavation, timely rock support was installed to follow the stand-up time rule for permissible limit for excavated rock mass. Stand-up time for rock support of excavated rock mass was taken from the similar underground cavern project which was excavated in granite gneiss terrain (Table 3).

Table3 Stand-up time chart for maximum allowable excavation limit for different rock type

Rock Class	I	II	III	IV	V
RMR (9InQ+44)	70~100	66~77	57~65	45~56	23~44
Suggested maximum unsupported span	40~50		30~20	12~15	1 or 2 round

7. Tunnelling methodology in adverse geological reaches:

During the excavation of pilot tunnel, controlled blasting with maximum pull 1.5 m to 2.0 m was taken in view of critical joints between chainage 113 m and 125 m (Fig 3). After excavation of each face, rock mass was supported with temporary rock bolts at the crown and walls above spring level. After the successful excavation of pilot tunnel, side slashing was very challenging because critical joints were extended/projected towards the end face of the cavern.

Figure 3 Joint 240-260°/65-70° exposed at chainage 113 m during pilot tunnel excavation

(i) Left side slashing: Left side slashing was done with controlled blasting from chainage 113 m and sufficient numbers of temporary rock bolts were installed in view of a critical joint S40-80°W/45-70° which was mapped during the pilot tunnel excavation. A tight detached rock block was observed along the joint at crown. Temporary rock bolts were installed in view of critical joint and projected its extension. Side slashing from left side was done upto138 m and then design rock support was completed. During the scaling work by boomer a rock chunk from critical joint was fallen on the boomer and minor damage was reported.

After this incident more precaution was taken during the side slashing work and after each blast careful assessment for temporary angle rock bolts were installed in view of persistence nature of joint. Left side slashing was done upto138 m.

(ii) Right side slashing: Right side slashing was started from chainage113 m with controlled blasting and temporary rock bolts support. After the each blast of side slashing sufficient numbers of temporary directional rock bolts were installed in view of critical joints. Design rock bolts were installed after the right side slashing upto 138m. Offset of

25-30 m was maintained between left and right side slashing and controlled blasting was adopted upto the chainage 175 m.

After 175 m chainage rock mass was observed slightly improved in this tunnel reach and no continuous joints were encountered. Only few numbers of temporary rock bolts were installed. Excavation between 175 and 210.6m was progressively faster in comparison of chainage between chainage 113 and 175m. After the left and right side slashing upto 210.6m all design rock bolts were installed.

During the excavation of pilot and side slashing work in these critical reaches drillability capacity of Boomer was fixed upto 2 m only to avoid any type of overloading of explosive.

Figure 4 Open joint at the right crown of pump house

(iii) Side slashing between 112 and 0.0 m: After rock bolt installation between 113 and 210.6 m of cavern heading portion area, side slashing for remaining unexcavated reaches between112 to 0.0 m chainage towards N 140° was started from right side slashing with temporary bolts. In this reach no persistence or critical joints were mapped during the pilot/side slashing work but difference between left and right side slashing were maintained 25-30 m throughout the side slashing. After completion of slashing upto83 m from right side, slashing from left side was started with temporary rock bolts. Pattern rock bolts were installed after every 25-30 m of right and left side slashing.

8. Rock support system:

After side slashing of heading portion all rock bolts were installed and installation map was prepared with the help of Surveyor total station equipment to detect the missing rock bolts, if any. All the pending rock bolts were installed to ensure stability of cavern before benching down excavation started**.**

To avoid displacement along joints, timely temporary and permanent rock bolts were installed within the standup time of rock mass. After the excavation of every 25-30 m of heading portion every 20 m tunnel area was supported with rock bolts. Joints were categorized into critical nature based on their orientation, persistence and filling along the joint.

9. Wedge analysis for adverse reaches of heading portion:

After the installation of pattern and additional rock bolts in the entire excavated area of pump house heading portion, Wedge analysis was carried out between chainage 113 and 175 m in view of sub horizontal joint N40°W-N80°W/5-30° critical joint S80°Wand N40°W/75-90° and S40-80°W/45-70°, joints were analysed in view of persistence, orientation and filling.

Average values of joints 300°/5° **(N60°W/5°)** 240°/55° **(S60°W/55°)** and 290°/80° **(N 70° W /80°)** were considered for the potential wedges. In this case roof wedge (7) was found with factor of safety 1.371 at crown (Fig.5). Design rock bolts and shotcrete thickness found sufficient for wedge formed area at the crown level (Fig.6).

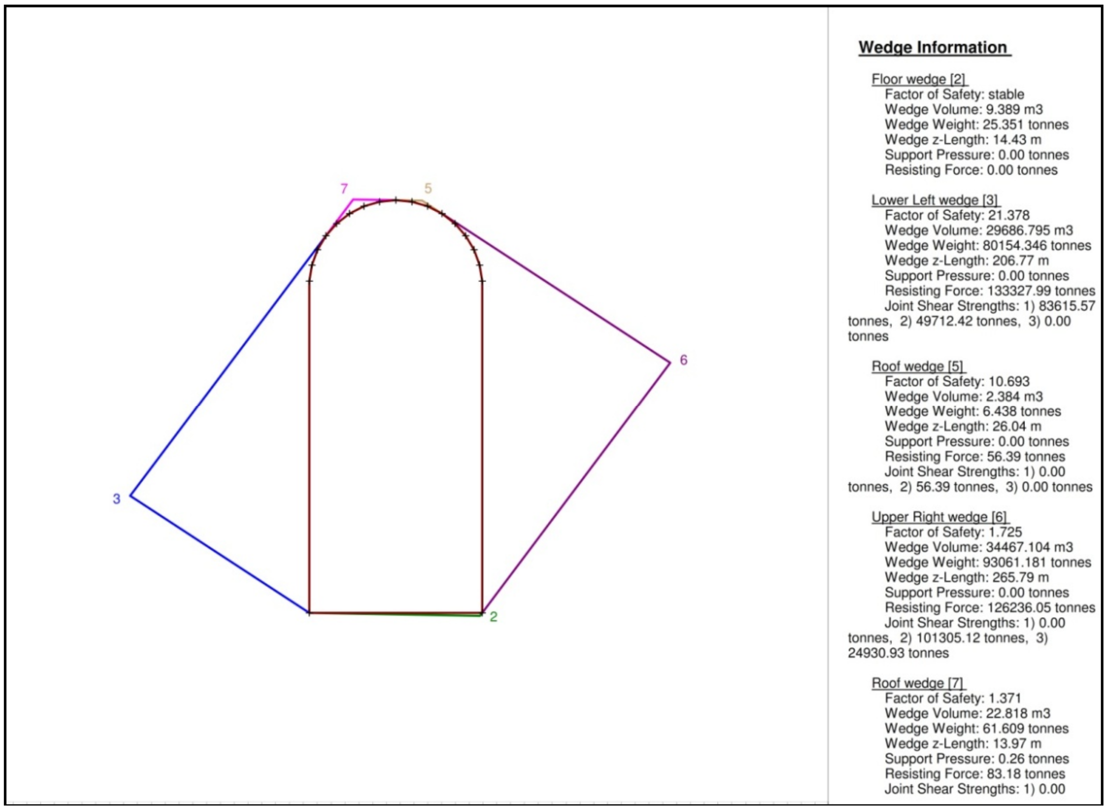

Figure 5 Roof wedge (7) formed due to sub horizontal joint 280-320°/5-30° with combination of other joints

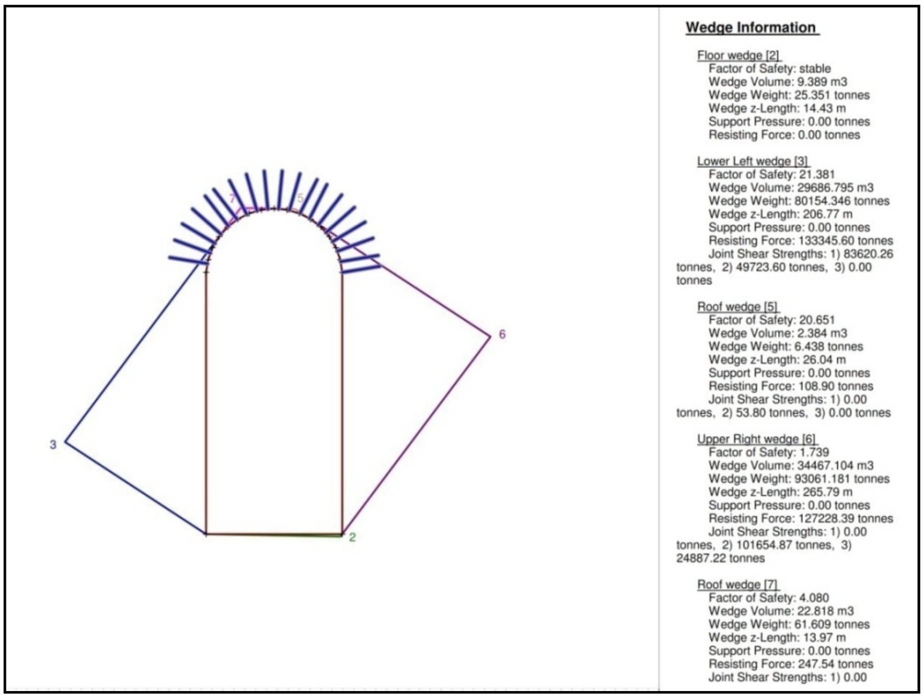

Figure 6 Factor of safety reached 4.080 after design rock bolts support for roof wedge (7)

Wedge analysis for joints 290°/80°**(N70°W/80°)** 240°/55° (**S60°W/55°**)and 080°/78° **(N80°E/78°)**were taken which showed wedge at roof level with factor of safety (0.000) factor of safety, which are below adopted factor of safety1.5. Analysis with support of pattern rock bolts and 110 mm thickness of shotcrete at crown, found factor of safety is 0.746 at crown. With installation of additional longer rock bolts 9-10 m at crown level factor of safety observed 2.164 and with design shotcrete thickness the factor of safety achieved upto 4.080 which found upto mark.

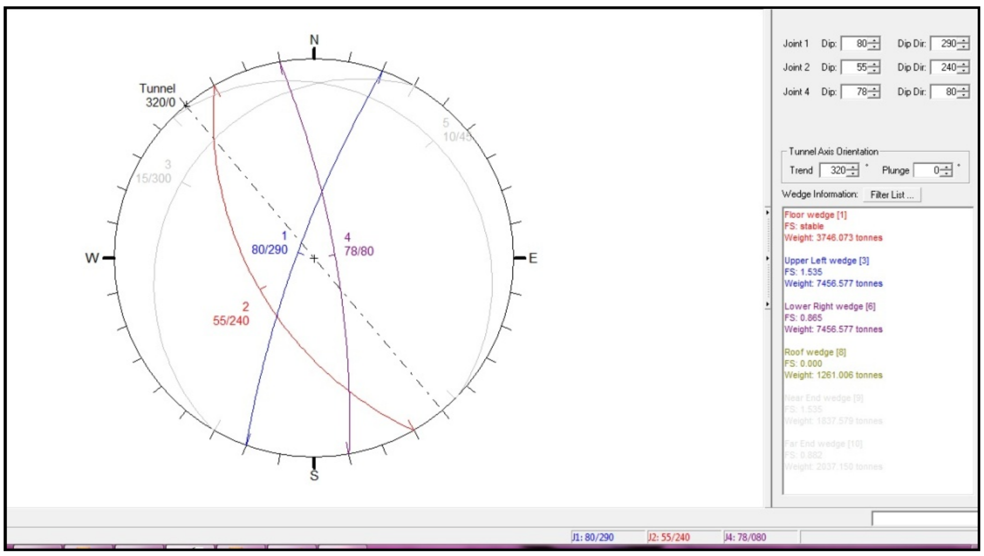

Figure 7 Roof wedge (8) due to intersection of critical joint N 260-320°/75-90° and other joints

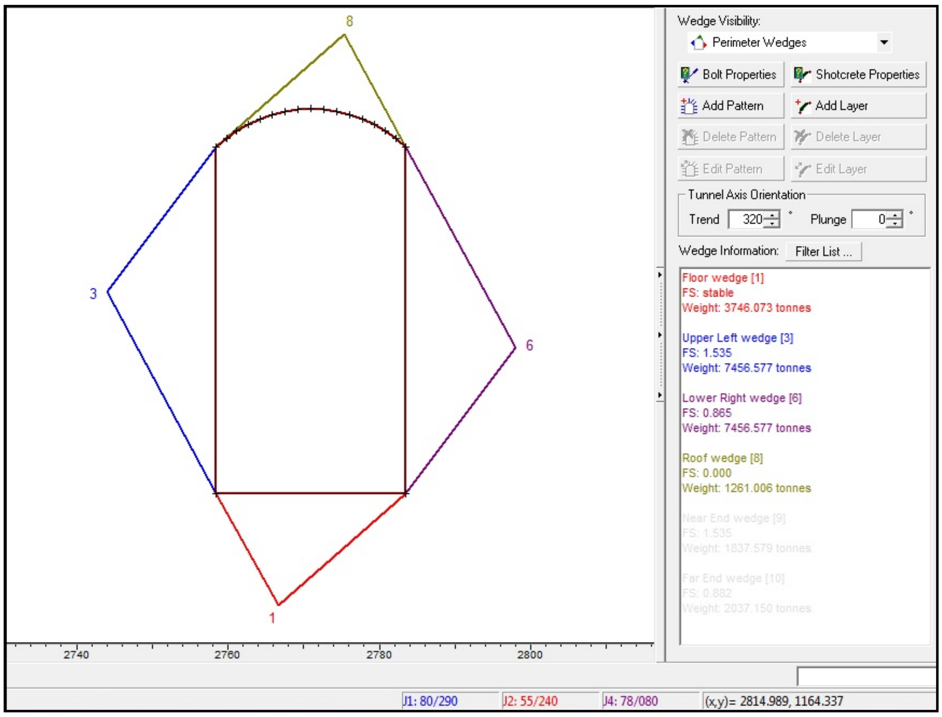

Figure 8 Potential wedge (8) information (F.S 0.000) due to intersection of joints

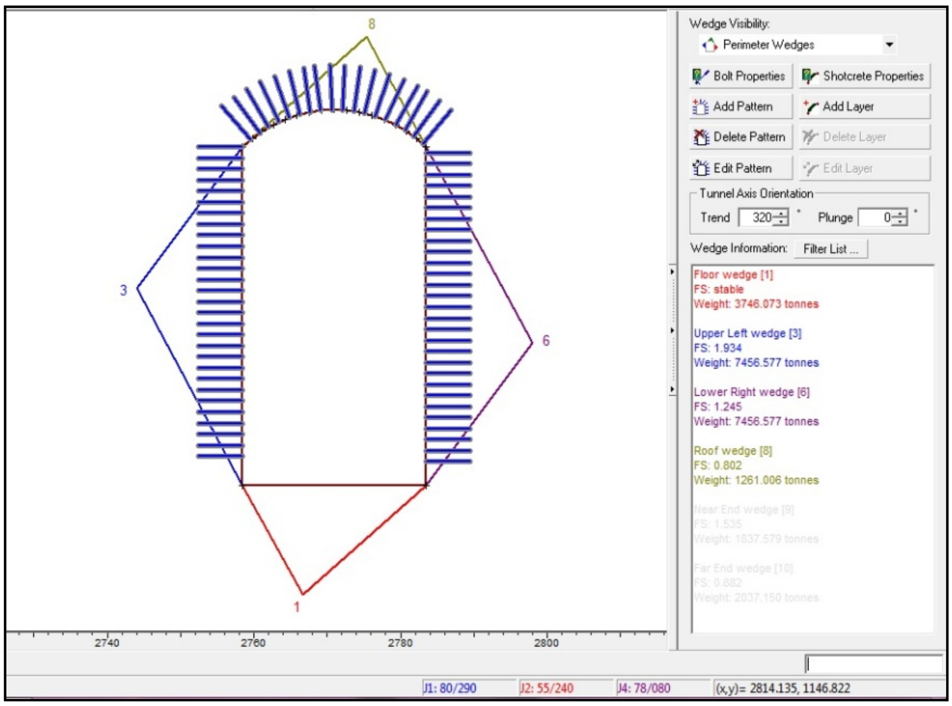

Figure 9 Potential wedge supported with design rock bolt and factor of safety is 0.748

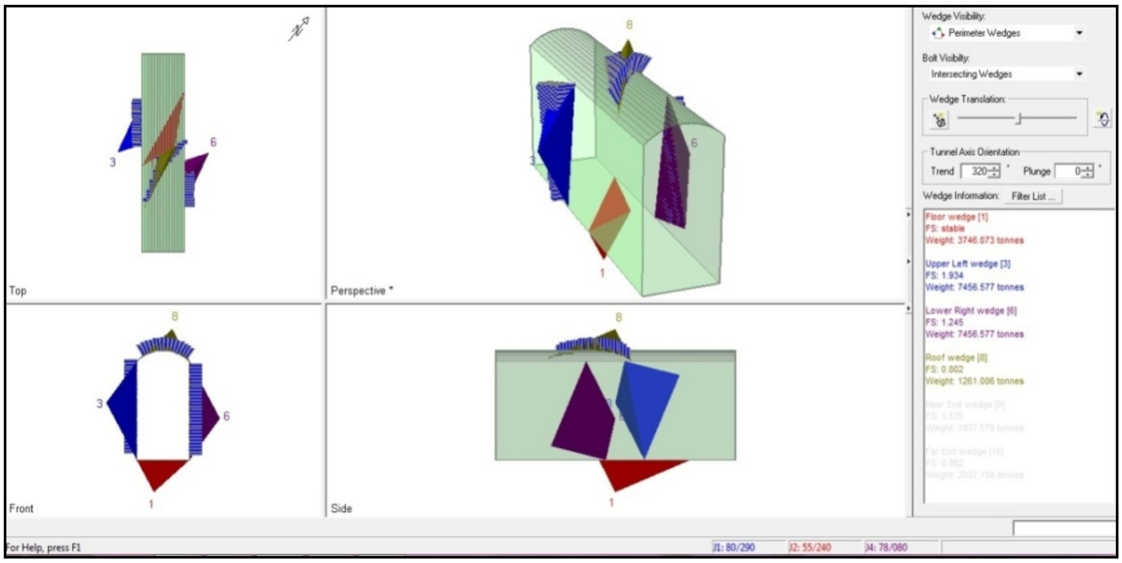

Figure 10 3D wedge view after installation of design rock support

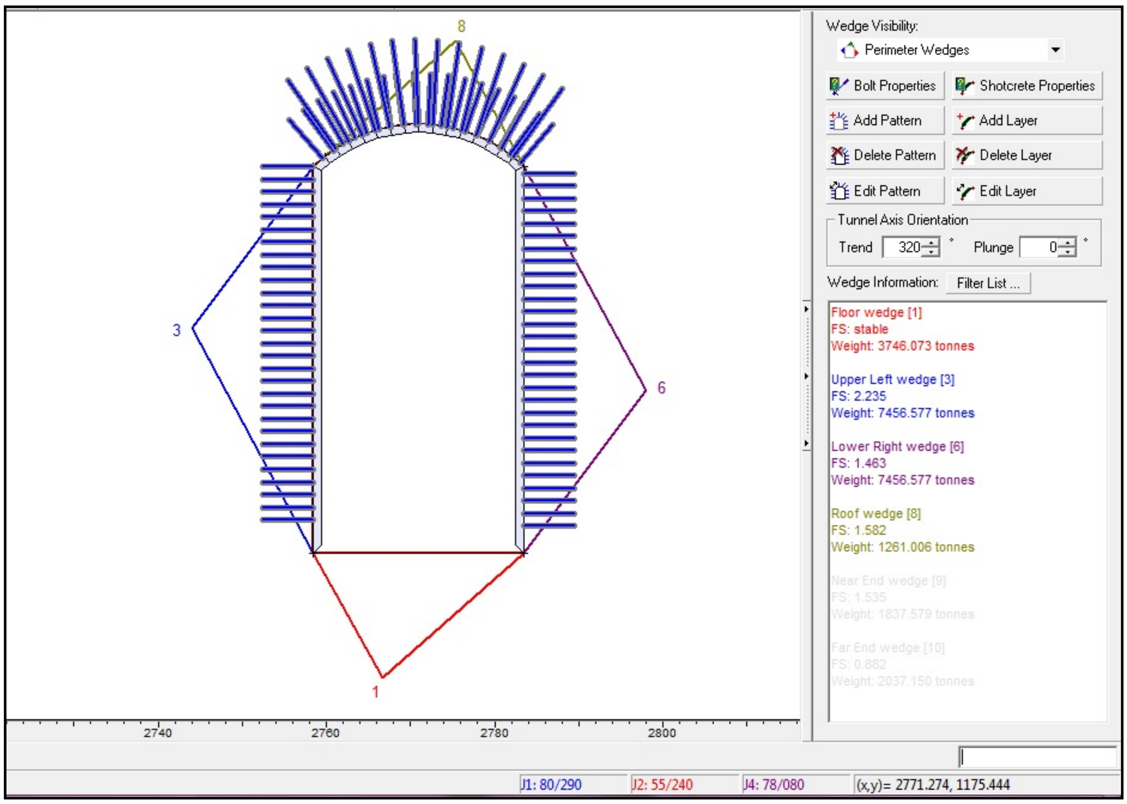

Figure 11 Roof wedge (8) supported with design and additional rock support and factor of safety upto 4.080

Joint combination	**Wedge type & support pressure**	**Volume & Weight**	**Joint shear strength**	**Safety factor without support**	**Design Rock Support**		**Additional rock support**		**Safety factor with rock support**	
					Rock bolt	**Shot crete thickness**	**Rock bolt**	**Shot crete thick ness**	**FS With Design support**	**FS With additional support**
J1-220-260°/45-70° J2-260-320°/75-90° J3-280-320°/5-30°	Roof wedge (7) Support pressure-0.26 tonnes	Volume-22.818 m3 Weight-61.691 tonnes	Joint shear strength-J1-0.00 tonnes J2-0.00 tonnes J3-83.18 tonnes	**1.371**	7 m long 25 mm dia at spacing 2 m C-C	110 mm at crown and 100 mm at walls	**No**	**No**	**4.080**	-
J1-220-260°/45-70° J2-260-320°/75-90° JR4-060-100°/75-80°	Roof wedge (8) Support pressure-9.06 tonnes	Volume-188.580 m Weight-509.166 tonnes	Joint shear strength-J1-0.00 tonnes J2-0.00 tonnes J3-0.000 tonnes	**0.000**	7 m long 25 mm dia at spacing 2 m C-C After rock bolt-Factor of safety-	110 mm at crown and 100 mm at walls	9-10 m long and 25 mm dia rock bolts	**50 mm**	**0.746**	**2.164** **After rock bolt 5.088**

Table 4 Detail of wedge information between chainage 113 and 175 m pump house reach

10. Conclusions:

Excavation methodology for critical reaches and recommendation of additional rock support system for treatment of adverse/critical geological features are always challenging. Identifications of adverse geological features, their projection for side slashing, benching down and additional rock support assessment are some important responsibilities during the excavation of pilot tunnel of any large underground cavern. Temporary and additional rock bolts lengths are established practices for treatment of geological features. Projection of adverse geological features/joints, prior to excavation reach of structure is always helpful in modifying the excavation methodology. This study shows that the experience gained during the excavation of pilot tunnel is very useful for side slashing, planning for rock support, necessary modifications and Unwedge analysis for critical joints.

Based on outcome of Unwedge analysis it was observed that potential wedge formation due to horizontal joint could be supported with design rock bolts but wedges formed by sub vertical critical joint, could be stabilized only with the additional longer rock bolts and shotcrete thickness.

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